Charles St to Washington St Drainage Improvement Study City of Harrisonburg

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Executive Summary

The residents of the neighborhood between Charles Street and Washington Street experience documented flooding. This drainage study was conducted to investigate the flood occurrences through an engineering-based flood study and investigate possible mitigation alternatives to alleviate frequent flooding. The total contributing drainage area through the neighborhood is approximately 490 acres, extending to the junction of Blacks Run at Monroe Street. The flood study performed drainage modeling which subdivided the 490 acre watershed into five sub-drainage basins to provide a detailed analysis of the runoff flow. The flow runoff from the sub-drainage basins were originally evaluated as part of an existing Federal Emergency Management Agency (FEMA) flow model (called HEC-HMS) for Blacks Run, which was used for establishing the FEMA regulated floodplain areas along Blacks Run. Further analyzing this watershed in greater detail than the FEMA study for the Blacks Run flow model (HEC-HMS) allows for an understanding of how this individual drainage basin contributes to the overall Blacks Run watershed, including understanding peak flow timing of this 490-acre drainage basin against the rest of the upstream Blacks Run watershed. The flood water elevations were determined using a 2-dimensional (2D) model developed by the U.S. Army Corps of Engineers (USACE) called HEC-RAS. HEC-RAS is a widely accepted flood modeling software for evaluation of flooding situations similar to this neighborhood. The flood water surface (HEC RAS) 2D model was built from scratch, specifically for this study. The calibration of the model was performed against the documented flood extents of the September 1, 2021 storm event, using actual rainfall data to recreate the flooding event in the model. Results of the calibration run were checked against photograph evidence of flood extent and time of extent. The results of this model run were considered comparable with the observed flood extents.

Multiple flood reduction alternative scenarios were modeled, and summarized below:

- Alternative 1: Upstream Stormwater Detention Options:
  - 1A: 30% flow runoff reduction against existing conditions in subbasin 2
  - 1B: 30% flow runoff reduction against existing conditions in subbasin 4
  - 1C: 30% flow runoff reduction against existing conditions in both subbasins 1 & 2
  - 1D: 50% flow runoff reduction against existing conditions in only the west basins (1, 2, 3)
- Alternative 2: Infrastructure Upgrades Only
  - 2A: Incorporating a concrete channel from Charles St to Ashby Ave, to facilitate greater flow conveyance
  - 2B: Concrete channel in 2A plus culvert upsizing under streets between Charles St to Ashby Ave
- Alternative 3: Combination of Detention and Infrastructure Upgrades
  - 3A: Combination of Options 1C + 2B

Results were evaluated against improvements to flood depth comparisons to home first floor elevations (FFE) located along the main flood path. Even with the best-case alternatives evaluated, flood reduction was not significantly improved over existing conditions. This analysis indicates significant flood improvement is constrained because of the study area is a low-lying/flat with limited capacity channels. Additionally, first floor elevations of dwellings are fixed and are not adequately elevated above flood limits. Furthermore, due to the relatively flat slope of the area, generating effective flow conveyance in additional pipe conveyance provides limited improvement. The detention alternatives were determined to be not viable due to the downstream impacts related to alignment of peak flow time of this watershed and that of Blacks Run. From a cost benefit analysis Alternative 2B shows the most cost-effective solution evaluated.

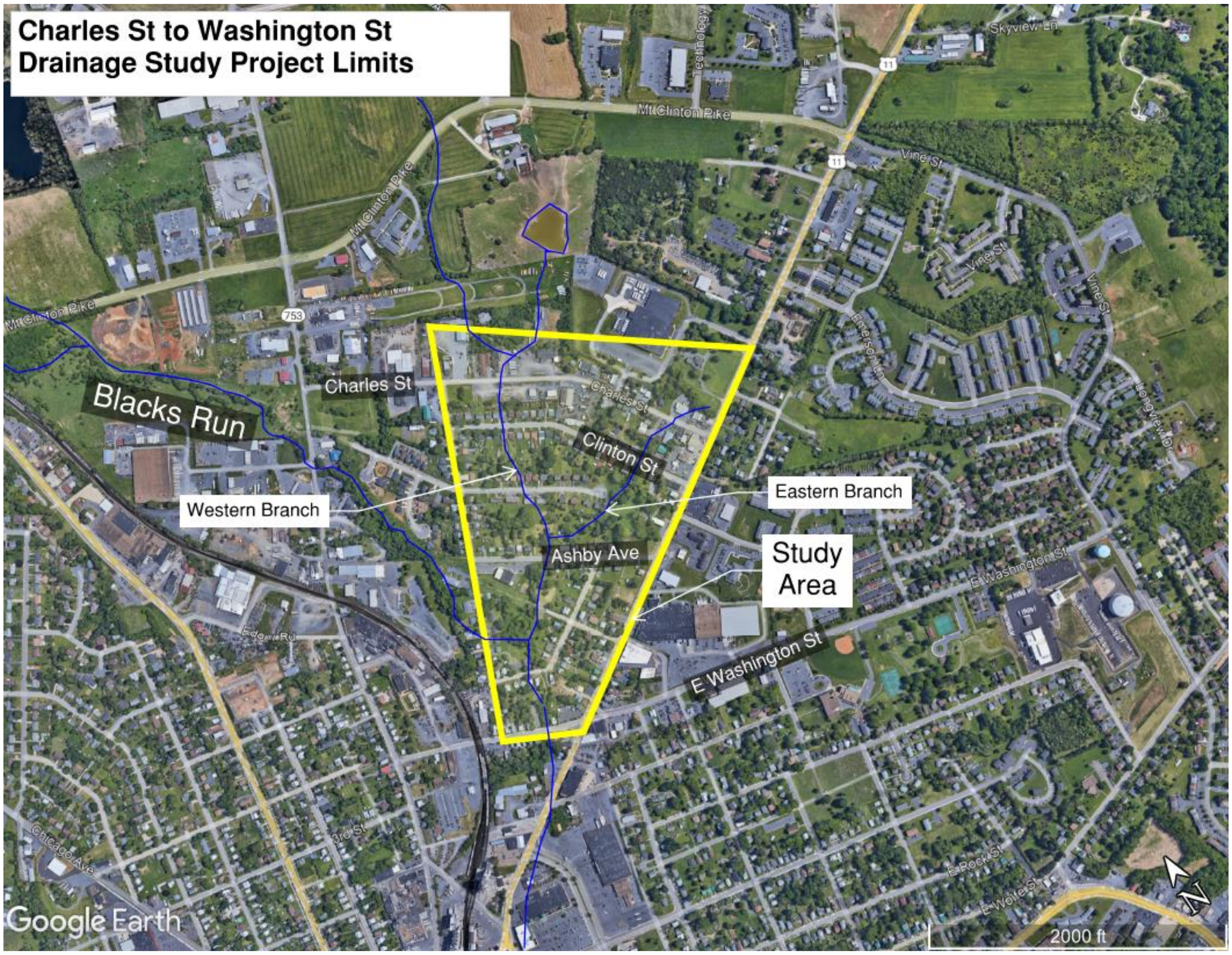

Figure 1: Map of Drainage Study Project Limits

1.0 Introduction

1.1 Background

Flood events within the drainage study project limits (Figure 1), a neighborhood in northeast City of Harrisonburg, Virginia (the City), have created flood occurrences for residents ranging from temporary road closures to property and structure inundation of varying degrees. Appendix A provides photo documentation for the August 31 to September 1, 2021, storm event flooding. The objective of this drainage improvement study is to investigate potential strategies to mitigate the flooding issues impacting the study area. Hydrologic and hydraulic analyses were conducted via HEC-HMS (version 3.5) and HEC-RAS (version 6.2) modeling software, respectively. This report details the analysis process, existing conditions evaluations, and resultant alternatives analysis.

1.2 Description of Basin Characteristics within Limits

The study limits (Figure 1) are bound by Jefferson St to the west, N. Main St to the east, Charles St to the north, and Washington St to the south. The area is part of a residential neighborhood consisting of mostly single-family homes (lots averaging approximately 10,000 square feet), a few townhome units, and commercial units. Storm runoff flow paths are produced along the lowest elevation terrain of the study area and join the main stem of Blacks Run between Monroe St and Madison St. The studies watershed is estimated to be approximately 0.76 square miles (490 acres) to the main stem of Blacks Run. The main stem of Blacks Run's drainage area is about 1.5 square miles at the Monroe St confluence. Drainage area maps are included in Appendix C.

Two main flow paths, which are separated by a ridge through the neighborhood, and are evident within the study area terrain. The western branch receives approximately 0.5 square miles of drainage and is formed by two smaller stream branches combining at the edge of the forested area upstream of Charles St. The documented flooding appears to be located along this western branch. The eastern branch, which originates from the northeast side Route 11, conveys a drainage area of about 0.2 square miles, and combines with the western branch in the private properties rear yard areas between Suter St and Ashby Ave.

Both the western and eastern flow paths are conveyed under the local streets and driveways via multi-line corrugated metal pipes (CMPs), shallow open channels through residential yards, and one concrete trapezoidal channel just upstream of Ashby Ave. A more natural stream channel is found downstream of Ashby Ave. Overland stormwater flows along street curb and gutters, entering the channels via inlets or curb cuts located at sag points. Two farm ponds and 12 various SWM BMP facilities are located within the drainage areas upstream of the study limits.

1.3 Site Visit and Survey

RK&K and WSSI conducted a site visit on May 9, 2022 to photograph existing site conditions and perform a preliminary survey of the existing hydraulic structures. During the May 9, 2022 field investigation, approximately 1 inch of cumulative rainfall fell within the previous 3 days, active flow was still observed through the field confirmed drainage channels locations. Site photos are included in Appendix A and survey data is summarized in Appendix B. The data collected from the field was used to confirm size, length, elevation and location of critical hydraulic structures for evaluation in the hydraulic model.

Overall, open channels are irregular, shallow (most less than 1 foot deep) vegetated channels with bottoms comprised of a gravel substrate. Ponding and frequent flooding of the adjacent yard areas was evident in the field study. All road culverts were observed to be elliptical CMP's, with minimal cover, and minimal pipe slope. The culvert end treatments are mainly concrete endwalls. Additionally, sediment accumulation was observed at several downstream ends of culverts, confirming a low energy system. In general, the drainage structures appeared to be in good condition and the vegetated areas within the study limits appeared to be stable, even after recent flow activity.

The field survey also captured the structural flow characteristics of the contributing drainage area just upstream and through the buildings found at 186 and 180 Charles Street. Field sketches and measurements of flow openings are documented in Appendix B. The flow was observed to flow under those buildings before entering the storm sewer system. The elevated buildings and structural supports act like box culverts. Observations from the site visit allowed more accurate hydraulic modeling of this section.

2.0 Hydrology

An existing hydrologic model developed with U.S. Army Corps of Engineers HEC-HMS software was updated to calculate discharges used as inputs for the subsequent hydraulic model. The existing HEC-HMS model was previously developed by another consultant for the 2008 FEMA study of Blacks Run, and last revised in 2019 for a FEMA Letter of Map Revision (LOMR) (effective date: March 26, 2020). RK&K updated the existing model to revise the subbasin (subbasin-164) characteristics documented in this reported study area. Specifically, revisions account for land use changes since the last model revision, as well as conform to current hydrologic standards such as the new National Oceanic and Atmospheric Administration (NOAA) Virginia rainfall distributions. Although newer versions of HEC-HMS are available, HEC-HMS version 3.5 was utilized during this project to resolve compatibility issues with newer versions of HEC HMS. Specifically, newer version of HEC HMS was searching for additional input data not used nor required for the original HEC HMS model of Blacks Run. For this study, the 2-year, 10-year, and 100-year 24-hour storms and an actual storm data for the August 31, 2021 to September 1, 2021 flood event were used for model validation.

2.1 Existing Hydrologic Model

Previously utilized to develop the flood analysis peak discharges for the Blacks Run FEMA study, the existing HEC-HMS model employed the Natural Resources Conservation Service (formally known as the Soil Conservation Service) (NRCS/SCS) Curve Number loss method and NRCS/SCS Unit Hydrograph transform method to compute watershed discharges. Reaches along Blacks Run and any modeled tributaries were routed via the Muskingum-Cunge method with representative channel parameters to simulate attenuation. Major reservoirs were also represented in the model, however, none that were included in the existing model directly impacted this project's study area. The entire study area was represented as Subbasin-164, a single 0.5 square mile watershed unit in the model.

The meteorological storm used in the original HEC HMS model used the outdated TP-40 rainfall and an NRCS Type II storm distribution to develop the 24-hour synthetic storm runoff. The TP-40 distribution has since been superseded with more localized rainfall depths and rainfall storm distribution, in the Virginia adopted NOAA Atlas 14 publication. A comparison of the estimated rainfall depths between TP-40 and NOAA Atlas 14 are shown below in Table 1.

Table 1: Comparison of Superseded TP-40 Rainfall Distributions with Current NOAA Atlas 14 Values

Storm Frequency	Precipitation Depth, 24-hour Duration (in)		% Change (%)
	TP-40, Type II	NOAA Atlas 14, Dale Enterprise Station	
2-year	3.4	2.62	-22.9
5-year	4.3	3.31	-23.0
10-year	5.2	3.89	-25.2
100-year	7.4	6.2	-16.2

2.2 HEC-HMS Model Updates

The original Subbasin-164 was updated in the RK&K model to replace the single subbasin-164, into five subbasins areas (Figure 2) to provide a great detailed analysis of this specific subbasin. Appendix C provides an aerial map of the delineated drainage areas. The reach between DA1/DA2 and DA3 (Reach-70) was modeled as a representative triangular channel and routed using the Muskingum-Cunge method. Inputs for this reach included length, slope, Manning's n, and side slope. The length and slope of Reach-70 were found based on the LiDAR data used in this study. The average reach slope was independently analyzed and updated in the analysis. The Manning's n value for this reach was derived from the HEC-RAS 2D User's Manual Table 2-1. and described as developed, medium intensity (Manning's n range from 0.08-0.16). Following standard practice, the average of the range was taken to obtain a final Manning's n value. Using a representative cross section (determined from LiDAR data), the average side slopes were calculated for the entirety of the channel. All design parameters for Reach-70 are summarized in Table 3.

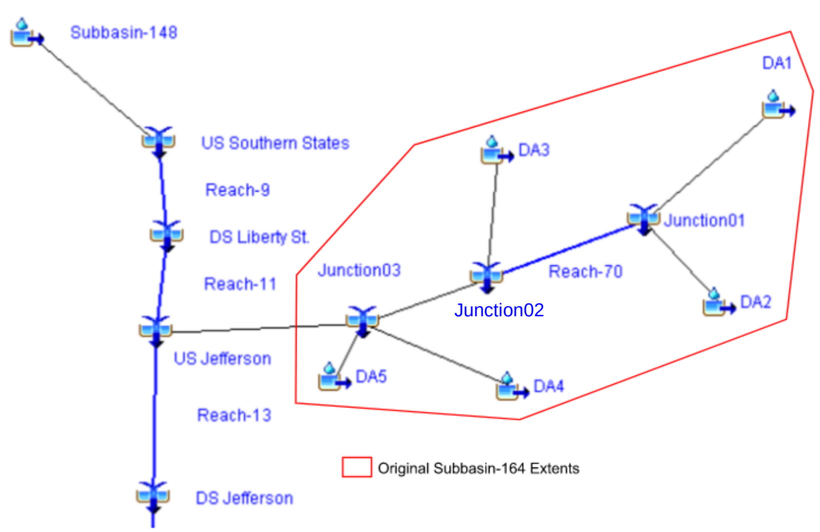

Figure 2: Schematic of HEC HMS subbasin 164 revision

Table 2: Comparison of Original vs. Updated HEC-HMS Model Hydrologic Inputs

Parameter	Original 2008 Study Subbasin	RK&K 2022 Updated Subbasin
Area (sq. mi.)	0.50	0.77
Curve Number, CN	71	76
Initial Abstraction (in)	0.82	0.82
Lag Time (min.)	54	53
Equiv. Tc (min.)	90	88

Table 3: HEC-HMS Inputs for Reach-70 Design

Parameter	Calculated HMS Input Value
Time Step Method	Automatic Fixed Interval
Length (ft)	1466.8
Slope (ft/ft)	0.0067
Manning's n	0.12
Shape	Triangle
Side Slope (xH:1V)	13.9

Data used to calculate watershed parameters included: the latest available GIS data of land cover, soils, and infrastructure from the City's GIS database (most layers last updated in 2018 or 2019), USGS bare earth digital elevation model (DEM) derived from 2020 LiDAR data, and 2020 aerial imagery of the City. The available land cover shapefiles were derived from aerial imagery, providing high-resolution detail. These shapefiles included roads and buildings, vegetated areas, and open land; they were regrouped via GIS tools into impervious cover, forest cover, and turf cover layers, respectively. All land cover layers were clipped to the external boundaries of each drainage area. The areas of impervious cover, forest cover, and turf cover were then calculated and assigned to standard hydrologic soil groups (HSG), soil type layer information was obtained from the United States Geological Survey- Web Soils Survey database. These areas were used to develop a weighted curve number (CN) based on NRCS TR-55 Table 2-2 for each drainage area, assuming good conditions for all areas (summarized in Table 2). For time of concentration calculations, overland flow, shallow concentrated, and channel flow types were all considered, when relevant, based on the topographical data. Overland sheet flow length assumptions were limited to 100 feet, as per NRCS recommendations.

The value for initial abstraction was assumed to be 0.82, to be consistent with the original 2008 HEC-HMS Model, which used initial abstraction to approximate initial losses attributed to karst formations. This approach is recommended and was developed in accordance with the Virginia Stormwater Management Handbook (Appendix 6-B) assuming 30% of the drainage area is karst.

The most significant update to the hydrologic evaluations, are related to correcting the watershed contributing drainage area from 0.50 square miles to 0.76 square miles based on this study's independent delineation. This increase in drainage area results in higher than previously analyzed peak discharges.

2.3 Analyzed Rainfall Depths

The 24-hour duration rainfall distribution depths were updated in the HEC-HMS model to reflect the NOAA Atlas 14 partial duration depths at the Dale Enterprise station (Site ID: 44-2208). Additional data is included in Appendix C.

Comparisons of the resultant peak discharges between this study and the previous analysis are included in Table 4 below.

Table 4: HEC-HMS Model Original vs. Updated Flow Results Summary

	2-yr		**10-yr**		**100-yr**	
Parameters	**Original 2008 Study Subbasin**	**RK&K 2022 Updated Subbasin**	**Original 2008 Study Subbasin**	**RK&K 2022 Updated Subbasin**	**Original 2008 Study Subbasin**	**RK&K 2022 Updated Subbasin**
Peak Outflow (cfs)	71.9	148.5	233	403.1	455.5	876.5
Total Outflow Volume Depth (in)	0.73	0.66	2.18	1.52	3.98	3.39
Time of Peak Flow (h:m from start of storm)	21:12	12:42	21:04	12:36	21:04	12:30

2.4 September 1, 2021 Storm Hydrology

For validation of the model, an actual storm event was simulated. Since photos of the resultant flooding at the study area were available, the August 31, 2021 to September 1, 2021 storm (September 1st storm) was selected. The closest rainfall gauge with available data was the Staunton, VA station (WBAN:93760). Incremental 20-minute rainfall depth measurements were obtained for this station and input into HEC-HMS as time-series precipitation data (Figure 3). Timesteps were set 1 hour ahead (gauge reported in local standard time (LST)) to align with the field observations, which were reported in daylight savings time (DST).

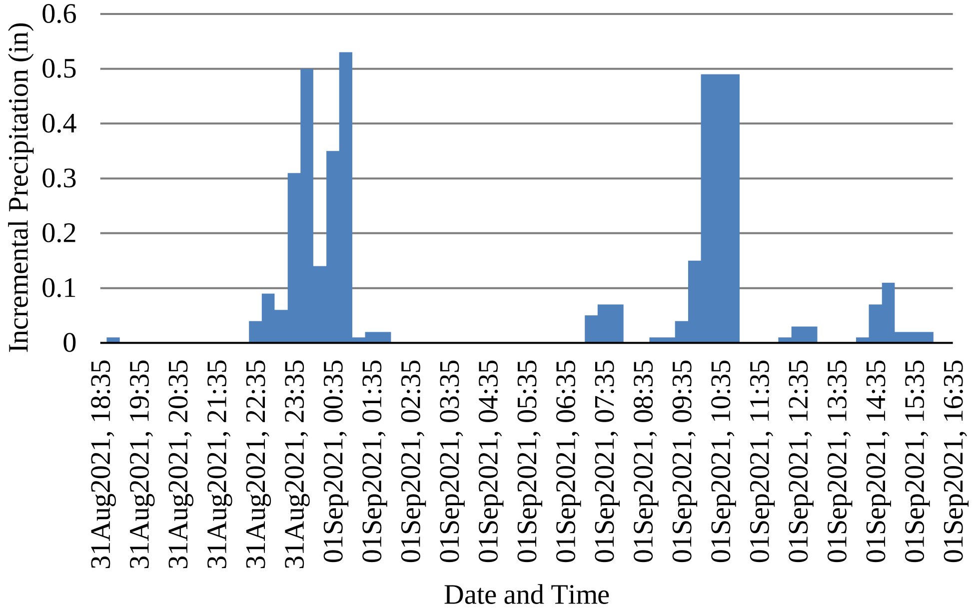

Figure 3: Aug. 31, 2021 to Sept. 1, 2021 Storm Precipitation Data (from Staunton, VA Station)

2.5 Hydrologic Results

Each storm event was simulated with a 6-minute computation interval in HEC-HMS assuming the same rainfall fell equally on each model subbasin. The simulation time was extended past the rainfall duration until the outflow receding limbs returned to 0 discharge. See Appendix D for graphical results and values.

Altogether, a contributing drainage area of 490 acres (0.76 square mile) travels through the study area. A significant volume of flow enters the study area from offsite, including DA1, DA2, and DA4. While a large amount of DA2 flows through a farm pond (the pond is assumed to provide little appreciable attenuation), DA1, DA3 and DA5 are largely uncontrolled. Appendix D provides the simulated results on a drainage area by drainage area basis. Resultant hydrographs for the main branches through the study area are also included in Appendix D. The western branch conveys about 72% of the entire contributing drainage area, and thus, about 70% of the result study area flow. The documented previous flooding occurrences are observed to occur along this flow path. Results are summarized in Table 5. Note during the 2-year, 24-hour storm, the peak flow through the western branch is almost 100 cfs greater than the eastern branch.

Table 5: Flow Results Through Each Concentrated Flow Path

Flow Path	Subbasins	Storm Frequency	Peak Discharge (cfs)	Time of Peak (from Start of Rainfall)	Volume (in)	Volume (ac-ft)
Western Branch	DA1,2,3	2-year	107.0	12:42	0.65	19.0
		10-year	286.8	12:36	1.51	44.1
		100-year	610.9	12:36	3.38	98.8
Eastern Branch	DA4	2-year	55.4	12:24	0.65	7.0
		10-year	145.0	12:24	1.51	16.2
		100-year	299.1	12:18	3.39	36.4

3.0 Existing Conditions Hydraulic Model

The U.S. Army Corps of Engineers HEC-RAS version 6.2 software was utilized to model the hydraulics within the study area. Due to the significance of overland flow to the flooding issues observed in the study area, as documented in past storm event photos, and observed in-the-field, an unsteady two-dimensional (2D) analysis of the study area was selected for this analysis.

An existing one-dimensional (1D) HEC-RAS model of the main Blacks Run reach modeled from about 2100 feet upstream of Mt Clinton Pike to about 700 ft downstream of I-81 was first developed for the 2008 FEMA flood study of Blacks Run, and last revised in 2019 for a FEMA LOMR (effective date: March 26, 2020). The new 2D model created for this study utilizes the results of the effective Blacks Run HEC RAS model as the assumed tailwater (downstream boundary) condition.

3.1 HEC-RAS Model Development

The 2D flow area perimeter was defined to encompass the project study limits and ensure that terrain elevations at the boundary are high enough, so no flow escapes the modeled area laterally. Upstream limits were delineated approximately 500 feet upstream of Charles St to allow for the upstream boundary condition areas to be accurately modeled. An upstream boundary condition was added for each main flow path through the study area and defined as inflow hydrographs. The downstream limit of the 2D flow area was delineated just upstream of where the tributary of the study area joins the main Blacks Run reach. Definition at this location enables the 1D model of Blacks Run to be independent from the 2D model, but remain linked hydraulically, as impacts of the main reach are experienced on the downstream end of the study area. The 2D model can then be easily adjusted to reflect any changes occurring within Blacks Run, which is separately enforced by FEMA floodplain studies.

Terrain used in the 2D model was the USGS bare earth DEM derived from 2020 LiDAR data. This terrain dataset was the most up-to-date readily available for the study area. In consideration of the significance of buildings’ impact on flow patterns as obstructions within the study area, building footprints were added to the base terrain file by raising DEM elevations 10 feet, the typical height of one story buildings. The building footprints were defined with an updated version of the City's buildings GIS shapefile layer (last updated in 2019); layer revisions were based on 2020 aerial imagery of Harrisonburg. Since the channels flowing through the 2D flow area are all intermittent streams and no base flow is present, the DEM data was assumed to adequately capture channel geometry. Therefore, no channel modifications were added to open channels. However, the terrain was modified in RAS-Mapper along culverts and storm sewer runs to model the storm sewer pipe infrastructure influence on flow patterns.

The City's land cover GIS shapefiles used to establish land cover conditions for the hydrologic model were also used to define the land cover conditions in the HEC-RAS model. Manning's n values (Chow, 1959) were assigned based on land cover type, see Table 6. Unpaved impervious surfaces were assumed to be gravel, paved driveways and sidewalks were assumed to be concrete, and paved parking, roads, and trails were assumed to be asphalt. Buildings were assigned an arbitrarily high Manning's n value to block flow through the structures. This 2D model conservatively assumes full saturation of pervious areas, which increases flooding likeliness. As a result, infiltrative properties were not considered for this 2D model. An example of this full saturation scenario is the September 1, 2021 storm used for validation, which experienced two intense rainfall periods during the total duration of the storm event. Initial abstraction would have been maximized early on during the first round of rainfall, then a second rainfall which generated the documented peak flooding.

Table 6: Manning's n Values Used

Surface Type	Manning's n Value
Paved Driveway	0.013
Paved Parking	0.016
Paved Road	0.016
Public Sidewalk	0.013
Trail	0.016
Unpaved Driveway	0.033
Unpaved Parking	0.033
Turf	0.03
Brush Outline	0.05
Crop	0.04
Tree Outline	0.1
Building	10

Existing storm sewer infrastructure was modeled as SA/1D connection culverts. The upstream Charles St building was modeled as a SA/1D connection bridge. Input dimensions and inverts were based on survey data from the May 9, 2022, site visit, GIS data, and DEM elevations. See Appendix E for details.

The unsteady model was run via the diffusion wave equations. This approximation of the full momentum equations is appropriate for this study because there is significant overland flow through the modeled area and channel flows are relatively shallow. Furthermore, since the diffusion wave method tends to overpredict diffusiveness, the method can account for the attenuation inherent in karst topography. The diffusion wave approximation also allows for a downstream boundary condition to be defined. This is beneficial for establishing an accurate, dynamic downstream boundary (tailwater) condition over time. This tailwater condition was characterized as a rating curve (elevation over time) at the downstream channel cross section, which was derived from unsteady 1D routing of the 24-hour storm events through the main Blacks Run reach (Appendix E Figure E22). At the upstream boundary conditions, HEC-HMS model results were input as 6-minute interval hydrographs. It should be noted the results will be overestimated at the upstream sides of these flow paths because the input flow represents the entire contributing area to the downstream end of the flow path. Since the flow path widths are relatively narrow (smallest single barrel culvert near upstream end has 36” diameter), in order to balance computation efficiency and detail, an overall cell size of 10’x10; was used, with refined cell sizes of 5’x5’ around structures and breaklines. Breaklines were delineated to fine-tune channel banks and allow model errors at abrupt elevation changes to be minimized. For model stability, a computation interval of 5 seconds was utilized.

Appendix E provides additional model results, were used to perform a sensitivity analysis of various modeling variables, to reach a conclusion of modeling result validation. The sensitivity analysis includes model runs which utilized the full momentum solver method and minimizing overland flow across the upstream areas of Charles St. Results of these model runs confirmed these scenarios did not significantly deviate from the final existing model results to justify incorporating into the effective existing model used for this study.

3.2 Summary of Existing Conditions

For model validation, the August 31, 2021, to September 1, 2021 storm event was routed through the HEC-RAS 2D model starting with hydrograph inflows at the upstream boundary conditions. The 2D simulation showed a rapid rise and fall of discharge through the watershed following each intense rainfall bursts.

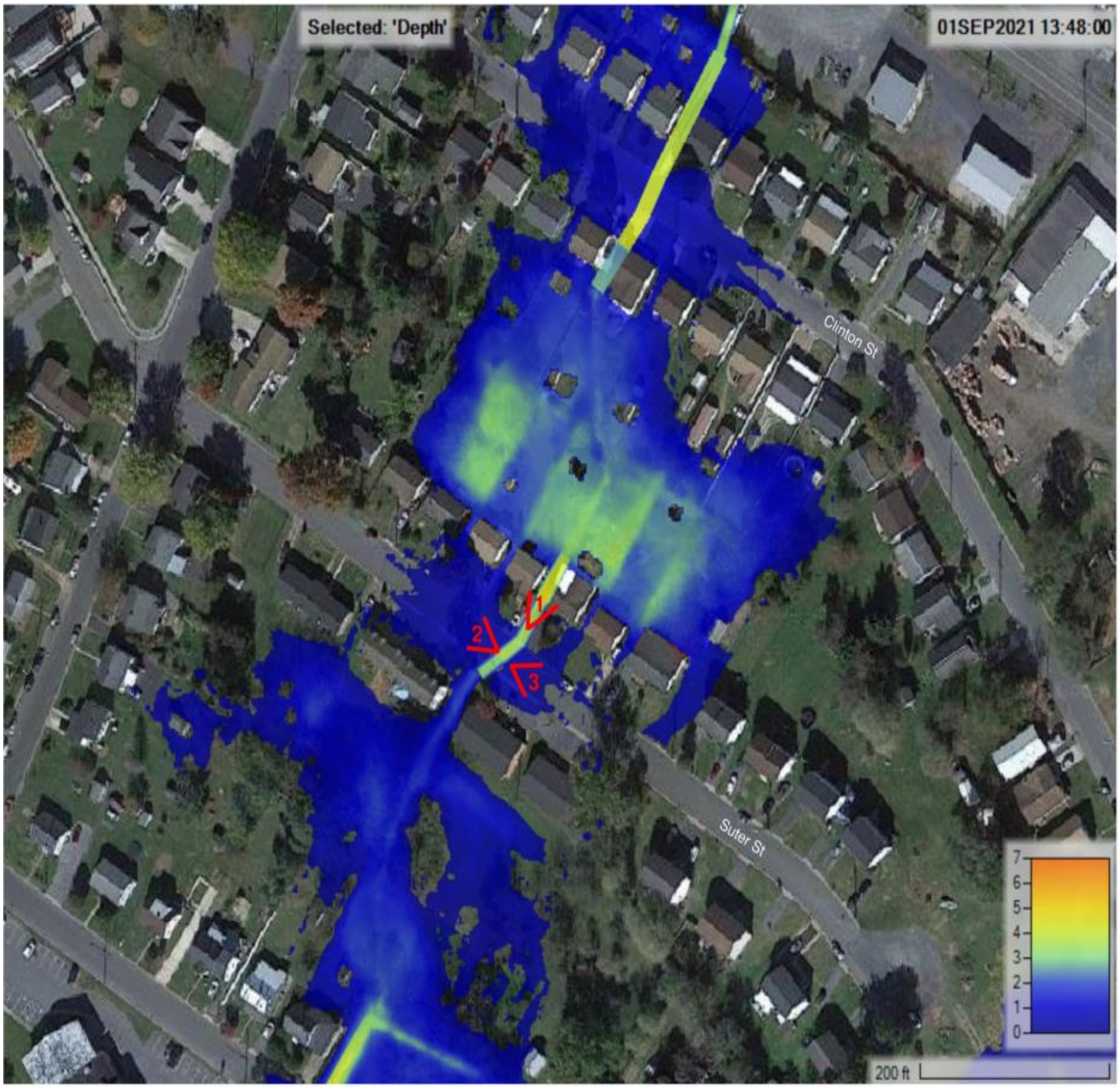

Figure 4: HEC-RAS 2D simulation of Sept. 1, 2021 storm. Corresponding event photos marked

Figure 5: Photo 1 depicts September 1, 2021, flood depths up to culvert headwall top located upstream of Suter St. Photo taken 9/1/2021 at 1:25PM.

Figure shows the modeled maximum flooding extents and depths that occurred due to the storm event. Visual comparison at various locations against photos of the resulting flooding provided by the City confirmed that overall, model results corroborated well with actual results. The photos were taken around 1:30 pm on September 1st, which captured instances of the recession after the storm's peak flooding, as a high water mark is visible on the buildings shown in 5 (Photo 1). The comparable HEC-RAS model snapshot was taken at a time of 1:48 pm, about 20 minutes after the storm's peak flooding, which occurs around 1:18 pm. This small timing discrepancy may be due to geographical timing differences between the rainfall data simulated (which was recorded in Staunton, the closest active rain gage) and the actual rainfall occurring on site. 5 (Photo 1) shows City personnel standing on top of the culvert headwall upstream of Suter St, with the water surface just below the top of the headwall. Based on standard VDOT EW-1A dimensions, the headwall should extend about 3.4' above the pipe invert. The modeled water depth at the headwall is also approximately 3.4' at the simulation timestamp.

Figure 6: Photo 2 depicts September 1, 2021, flood extents on Suter St. on right side of the culvert outlet (facing downstream). Photo taken on 9/1/2021 at 1:28PM

Figure 7: Photo 3 depicts September 1, 2021, flood extents on Suter St. on left side of the culvert outlet (facing downstream). Photo taken on 9/1/2021 at 1:30PM.

Figure 6 (Photo 2) validates the flooding extents occurring around the right edge of the townhouse building on the right side of the culvert outlet (facing downstream). On the left side of the culvert outlet (facing downstream), the modeled flooding extends just past the left edge of the brick building and the electric pole on the upstream side of the road. A similar scenario can be seen in Figure 7 (Photo 3), but the edge of flooding does not extend as far left of the culvert, indicating that there is a small overestimation in flooding extents on the left side of the culvert.

Maps in Appendix F show the modeled extents and depths of the 2-yr, 10-yr, and 100-yr 24-hour storms. Modeling confirmed the low-lying areas in backyards experienced the deepest ponding/flooding depths. A key area of concern are the backyards between Clinton St and Suter St, where widespread flooding was documented in the modeling results. During the 2-yr storm model simulation, ground floors (first floor elevations-FFE) of properties likely get flooded. Further downstream, modeling results indicate flooding occurs in the backyards of the properties adjacent to and downstream of the Suter St culvert crossing, though it appears the first floor elevations (FFE) of these properties remain above flood surface elevations. Significant street flooding is shown in the modeling to occur at the road low points on Clinton St and Suter St during the simulated 2-yr storm event. Ashby Ave experiences road flooding during the modeled 10-yr storm. See Table 7 in Appendix F for modeled maximum depths and estimated impacts on various buildings along the flow path.

Table 7: Summary of Maximum Depth Results

Storm	Max. Depth (ft)	Location of Max. Depth
2-year	4.0	D/S end of Ashby Ave concrete channel
10-year	4.7	D/S end of Ashby Ave concrete channel
100-year	5.0	D/S end of Ashby Ave concrete channel

Model results also confirm undersized storm sewer infrastructure performance. See Table 8 for a summary of results. Storm pipes in the City of Harrisonburg are intended to be designed for the 10-year storm. However, storm sewers within the study area were constructed before current drainage design standards were adopted. During the 10-year storm modeling, all except the upstream storm pipe connecting to the Suter St crossing are overtopped. Between Suter St and Clinton St, the downstream ponding causes backwater conditions upstream flow through the pipes.

Table 8: Summary of Simulated Culvert Performance

2-year 24-hour Storm						
Structure	**Total Flow**	**Total Culvert Flow***	**Total Weir Flow***	**HW**	**TW**	**Notes**
	(cfs)	**(cfs)**	**(cfs)**	**(ft)**	**(ft)**	
Charles Bldg 1	1.65	-	-	1350.93	1348.18	no overtopping
Charles Bldg 2	29.4	19.83	-9.57	1348.21	1348.02	
Charles Storm US	21.33	21.33	0	1348.03	1347.62	
Charles Storm DS	26.05	21.39	4.66	1346.95	1346.21	
Clinton St	50.58	-48.97	-1.61	1346.44	1347.17	backwater impacts
Clinton St DS	68.08	-68.08	0	1344.78	1347.48	backwater impacts
Suter St US	30.19	-30.19	0	1344.04	1345.1	backwater impacts
Suter St culvert	30.33	27.62	2.71	1343.83	1342.65	
Ashby Ave culvert	96.9	96.4	0.5	1339.94	1338.95	
Monroe/Greenway	27.51	3.56	23.95	1336.06	1336.05	
10-year 24-hour Storm						
Structure	**Total Flow**	**Total Culvert Flow***	**Total Weir Flow***	**HW**	**TW**	**Notes**
	(cfs)	**(cfs)**	**(cfs)**	**(ft)**	**(ft)**	
Charles Bldg 1	1.66	-	-	1350.66	1348.07	no overtopping
Charles Bldg 2	48.87	33.96	-15.23	1349.43	1349.04	
Charles Storm US	43.81	41.03	-0.08	1349.42	1348.05	
Charles Storm DS	28.94	21.28	-2.18	1347.02	1346.28	
Clinton St	49.42	-50.12	-0.42	1346.36	1347.13	backwater impacts
Clinton St DS	72.15	-71.57	-7	1344.94	1347.72	backwater impacts
Suter St US	38.01	-37.53	0	1344.28	1345.9	backwater impacts
Suter St culvert	31.06	27.67	2.57	1343.81	1342.63	
Ashby Ave culvert	110.65	99.75	11.82	1340.21	1339.14	
Monroe/Greenway	89.42	7.21	69.04	1336.4	1336.33	

100-year 24-hour Storm						
Structure	**Total Flow**	**Total Culvert Flow***	**Total Weir Flow***	**HW**	**TW**	**Notes**
	(cfs)	**(cfs)**	**(cfs)**	**(ft)**	**(ft)**	
Charles Bldg 1	2.62	-	-	1350.93	1348.22	no overtopping
Charles Bldg 2	43.58	30.44	-13.14	1349.06	1348.72	
Charles Storm US	58.38	43.56	14.82	1350.02	1348.46	
Charles Storm DS	25.26	21.08	4.18	1347.02	1346.29	
Clinton St	49.9	-49.9	0	1346.31	1347.07	backwater impacts
Clinton St DS	63.89	-63.89	0	1345.26	1347.42	backwater impacts
Suter St US	39.85	-39.85	0	1344.59	1346.41	backwater impacts
Suter St culvert	31.11	27.67	3.44	1343.85	1342.67	
Ashby Ave culvert	113.7	100	13.7	1340.25	1339.18	
Monroe/Greenway	142.58	5.24	137.34	1337.07	1337.04	

* Negative flows indicate upstream flow direction

4.0 Alternative Analysis

The existing model was updated to analyze various potential design solution alternatives. These alternative solution scenarios were compared to determine relative effectiveness of addressing modeled flood conditions in the existing condition. Effectiveness of each alternative was evaluated primarily based on the reduction of flood depths against surrounding buildings adjacent to the western branch, where most of the flooding is observed, relative to first floor elevations (FFE). The maximum flood extents (plan view), overall maximum flood depths, and downstream impacts were also evaluated. The following alternative scenarios were modeled:

- Alternative 1: Upstream Stormwater Detention Options:
  - 1A: 30% flow runoff reduction against existing conditions in subbasin 2
  - 1B: 30% flow runoff reduction against existing conditions in subbasin 4
  - 1C: 30% flow runoff reduction against existing conditions in both subbasins 1 & 2
  - 1D: 50% flow runoff reduction against existing conditions in only the west basins (1, 2, 3)
- Alternative 2: Infrastructure Upgrades Only
  - 2A: Incorporating a concrete channel from Charles St to Ashby Ave, to facilitate greater flow conveyance
  - 2B: Concrete channel in 2A plus culvert upsizing under streets between Charles St to Ashby Ave
- Alternative 3: Combination of Detention and Infrastructure Upgrades
  - 3A: Combination of Options 1C + 2B

4.1 Alternative 1: Upstream Stormwater Detention Only

Alternatives 1A, 1B, 1C

Detention only alternatives were evaluated for targeted stormwater management approaches in sub basins DA2 (Alt 1A), DA4 (Alt 1B), and combination DA1 &DA2 (Alt 1C). The potential detention facility locations considered include specifically the existing farm pond in DA2 which would be retrofitted, the parcel near the intersection of N Main St and Charles St in DA4, and the bus parking lot owned by Rockingham County in DA1 (Figure 8).

Hydrologic parameters were estimated for the alternative evaluations. See Table 9 for the assumed subbasin adjusted conditions used for the alternative 1 scenario. To simulate alternatives detention performance, the drainage area and time of concentration of each subbasin was estimated to be reduced by 30% and peak time concentration was extended an additional 20 minutes, respectively, to replicate the reduction in peak flow and extending of the hydrograph peak timing.

Table 9: Alternative Subbasin Parameters to Simulate Detention Facility Impacts

Parameter	**Subbasins**				
	DA1	**DA2**	**DA3***	**DA4**	**DA5***
Proposed Facility Location	Rockingham County Bus Lot	Existing Farm Pond	-	N Main St/Charles St Parcel	-
Area (sq. mi.)	0.1693 (0.08465)	0.1735 (0.08675)	0.0582 (0.0291)	0.1410	0.0162
Curve Number	73	77	83	76	80.3
Initial Abstraction (in)	0.82	0.82	0.82	0.82	0.82
Lag Time (min.)**	50.9	33.4	13.1	31.8	11.1
Equiv. Tc (min.)	84.8	55.6	21.8	53	18.5

*All drainage area properties remain same as existing conditions
(DA for Alternative 1D) ** Lag time increased by 0.6(20 min.) = 12 min.

Note: The constructability of these potential SWM facilities depends on site characteristics. For example the geometry would be limited by the parcel’s available surface area, any existing utility conflicts, inverts of nearby existing storm infrastructure, and design criteria.

Figure 8: Location of Proposed Detention Facilities

Alternative 1D

The effect of a new stormwater management facility upstream of the study area was analyzed by reducing the overall flowrate through the western branch (DA1, DA2, & DA3) by 50%, without any detention time adjustments. A 50% flowrate reduction approaches the maximum reduction that can be reasonably achieved with a detention facility and is often unfeasible due to space limitations. However, modeling this significant amount of flow reduction enables evaluation of the maximum benefits of a detention only alternative.

Results of Alternative 1A, 1B, 1C, & 1D

Appendix H1 provides detailed flood comparison mapping and resultant changes in flood levels against houses located adjacent to the primary flood path in the Western branch. The results demonstrate that the result of stormwater detention and reduction of peak flow through individual areas or collectively throughout the western branch did not all eliminate flooding within the study area, especially during lower return period storms (storms greater than 2-year storm). Below is a summary of the results for the four Alternative 1 scenarios. Alternatives 1A, 1B, and 1C, are the most feasible, but yielded minimal improvements. Alternative 1D, did demonstrate a significant improvement for the 2-year storm, by removing all but one house from flooding the first-floor elevation, but during the 10-year storm only 2 of 7 flooded houses would improve to a point which eliminates first floor flooding. The results suggest the upstream detention will at best only mitigate flooding during the 2-year storm and improvements in infrastructure would be necessary in order to achieve flood mitigation goals.

Alternative		Results		
		2-year, 24-hour Design Storm	10-year, 24-hour Design Storm	Sept. 1, 2021 Storm Event
1- UPSTREAM STORMWATER DETENTION FACILITIES ONLY	A	Overall ~0.1' drop in flood WSE's relative to existing conditions, but small increases around Ashby Ave. Slight decrease in planar flood extents along western branch.	Overall ~0.1' drop in flood WSE's relative to existing conditions. Slight decrease in planar flood extents along western branch.	Overall ~0.15' drop in flood WSE's relative to existing conditions. Slight decrease in planar flood extents along western branch.
	B	Small drop in flood WSE's around Ashby Ave and downstream. Slight decrease in planar flood extents along eastern branch and downstream of Ashby Ave.	Very minor decreases in flood WSE's around Ashby Ave and downstream. Slight decrease in planar flood extents along eastern branch.	Less than 0.1' drops in flood WSE's around Ashby Ave and downstream. Slight decrease in planar flood extents along eastern branch.
	C	Drops flood WSE's between 0.14'-0.33' relative to existing conditions, lowers all but one flood WSE's below building FFE's. Noticeable decrease in planar flood extents along western branch.	Drops all flood WSE's between 0.04'-0.41', eliminates FFE flooding of 1 of 7 buildings flooded during existing conditions. Significant decrease in planar flood extents along western branch.	Drops all flood WSE's between 0.1'-0.57', Eliminates FFE flooding of 2 of 8 buildings flooded during existing conditions. Significant decrease in planar flood extents along western branch.
	D	Drops flood WSE's between 0.18'-1.6' relative to existing conditions, lowers all but one flood WSE's below building FFE's. Significant decrease in planar flood extents along western branch.	Lowers all flood WSE's between 0.14'-0.67', eliminates FFE flooding of 2 of 7 buildings flooded during existing conditions. Significant decrease in planar flood extents along western branch.	Drops all flood WSE's between 0.5'-0.76', eliminates FFE flooding of 2 of 8 buildings flooded during existing conditions. Significant decrease in planar flood extents along western branch.

4.2 Alternative 2: Proposed Storm Sewer Infrastructure Improvements Only

Alternative 2 evaluated two optional scenarios for storm sewer infrastructure improvements.

Alternative 2A

Alternative 2A evaluated creating well defined concrete channels between the streets to promote greater flow capacity through the backyard areas.

Iterations of improvements were analyzed from least invasive improvements to most invasive. First, a representative cross section was developed that would provide adequate capacity to convey the flow through the western branch. The trapezoidal concrete channel is proposed to have a 6' wide bottom width with 2:1 side slopes and a minimum of 2' depth, with a minimum longitudinal slope of 1% and Manning's n roughness of 0.013. For a slope of 0.5%, which occurs between Clinton and Suter St, the bottom width was expanded to at least 8 ft to have a similar conveyance capacity. Adding a manmade concrete channel alone helped mitigate flooding between Suter and Ashby during the 2-yr storm. However, the open area between Clinton and Suter remained flooded, which strongly indicated that the culverts crossing at Suter St are undersized and cause significant backwater effects. In fact, a test with the Suter St culverts removed and just modeled as an open channel resulted in significant flood reduction in the yard area, as pictured in Figure 9. However, a road crossing is necessary and an open channel in this location is not feasible.

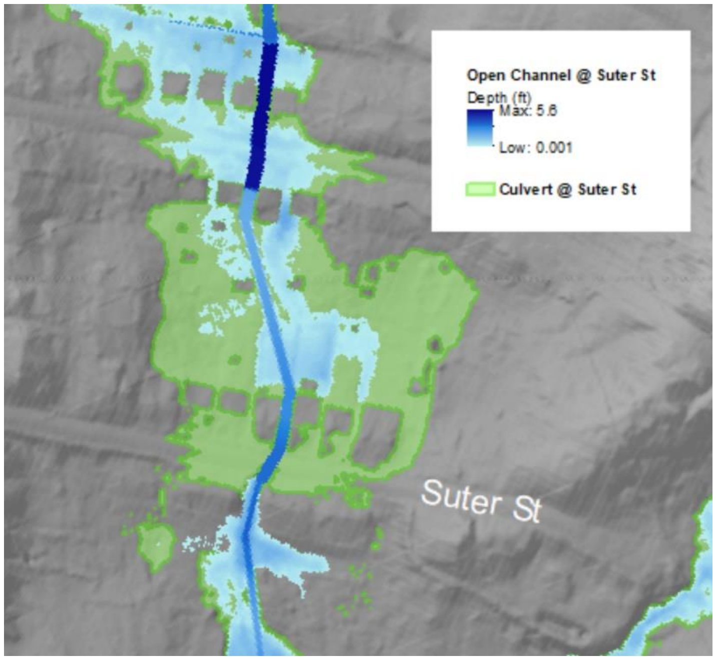

Figure 9: Open space flooding reduced when Suter St culverts converted to open channels

Alternative 2B

Alternative 2B evaluated taking the Alternative 2A channels and adding improved street storm sewer infrastructure (i.e. existing culverts) with larger pipes.

Street infrastructure improvements focused on upsizing the existing storm systems. Systems were designed for the 10-year storm where possible. Multiple barrel runs, with lowered inverts to provide adequate cover, were proposed. At Suter St and Ashby Ave, a triple and quadruple 5'x3' concrete box culverts were evaluated, respectively, rather than elliptical pipes to help maximize flow capacity while minimizing required space. The storm sewer system outfalling downstream of Clinton St was upsized to accommodate the 10-year design storm, which was achieved with 4 barrel 45"x29" elliptical concrete pipe runs, with about a 0.6% slope at minimum. 2'x2' ditches were also added along a section of Charles St to bring water ponded in the sag area into the storm system. The alternative HEC-RAS inputs are included in Appendix G.

Results of Alternative 2B

Flood water surface elevation (WSE) results due to infrastructure changes (Tables H4.1-H4.3 in Appendix H) vary from area to area. At most of the locations measured, flood WSE's were below existing condition flood WSE's. The 2-year storm stimulations with infrastructure improvements resulted in lowered flood elevations at the buildings measured except at House A, although still below the first-floor elevation. This pattern did not persist during the 10-year and September 1st storm simulations and rather, an increased flood elevation relative to existing conditions was measured at Lot K and House J, respectively. These inconsistencies may be attributed to modeling error. While infrastructure improvements effectively eliminated building flooding for all but one building analyzed during the 2-year storm event, flood elevation reductions were not enough to completely eliminate flooding of the buildings analyzed during the more intense storm scenarios modeled. Maps of the resultant flood extents are included in Appendix H.2. Results show significant flood reduction for a 2-year, 24-hour storm event, especially on the streets and in the open area between Suter St and Ashby Ave. While the open area between Clinton St and Suter St is still flooded, the flooded area has been reduced. However, not much planar (horizontal flood extent) improvement is visible during simulations of the 10-year, 24-hour storm event or the September 1st, 2021 storm, as the roads still exhibit flooding. Also, some increases in flood extents are observed downstream of Ashby Ave due to the higher capacity pipes being able to convey more concentrated flow more rapidly downstream. Overall, infrastructure only improvements demonstrated reasonable improvements to mitigating flooding in the study area caused by smaller frequency storms such as the 2-year, 24-hour event, but similar to detention only (alternative 1) alternatives, are less effective during the 10-year and September 1, 2021 events.

Alternative		Results		
		2-year, 24-hour Design Storm	10-year, 24-hour Design Storm	Sept. 1, 2021 Storm Event
2- STORM SEWER INFRASTRUCTURE IMPROVEMENTS ONLY	A	Lowers most flood WSE's between 0.05'-0.4' except around buildings along Suter St. Reduces planar flood extents in open space areas and along the roads.	Lowers all flood WSE's between 0.04'-1.21', eliminates FFE flooding of 2 of 7 buildings flooded during existing conditions. Reduces planar flood extents in open space areas and along the roads.	Lowers most flood WSE's between 0.05'-0.25' except around buildings along Suter St. Reduces planar flood extents in open space areas and along the roads.
	B	Lowers all but one flood WSE's below building FFE's, eliminates 2 of 3 flooded buildings previously measured at existing conditions and eliminates abutting floodwater from 5 buildings previously measured at existing conditions. Significantly reduces planar flood extents in open space areas and eliminates most flooding along the roads.	Lowers all flood WSE's between 0.04'-1.26', eliminates FFE flooding of 1 of 7 buildings flooded during existing conditions and eliminates abutting floodwater from 1 buildings previously measured at existing conditions. Reduces planar flood extents in open space areas and along the roads.	Lowers all flood WSE's between 0.1'-0.3'. Reduces planar flood extents in open space areas and along the roads.

Note: there are challenges to improved infrastructure including minimal cover over culverts (i.e. depth between top of pipe and road surface), narrow open spaces between houses, and relatively flat elevations throughout the study area to allow for pipes to be placed deeper.

Alternative 3: Combination of Detention and Storm Sewer Infrastructure Improvements

Results of Alternative 3A

Overall, flow reductions help reduce the lateral extents and vertical depths of flooding during the 10-year storm, as observed in the Appendix H.3 maps. Though the timing of the flow peaks significantly impact results locally. Some buildings that did not experience flooding during the improved infrastructure scenario exhibited elevated flood levels when detention was added.

Flow peaks at large junctions within the study area, from the various detention scenarios proposed, were compared with their respective discharges from existing conditions. The hydrographs included in Appendix G, the inflow from the study area at the Blacks Run junction peaks during the rising limb of the Blacks Run hydrograph, and relatively close to the Blacks Run peak time. If a detention design generates prolonged, elevated discharges through the study area in a manner that coincides with the flow peak of Blacks Run, flooding increases would occur in Blacks Run. Therefore, any detention measures implemented that impact the study area must also be analyzed downstream to ensure that the release of flows do not cause detrimental flood impacts to the downstream end of the study area or Blacks Run. Further hydrologic analysis showed that despite reducing flooding in upstream areas, detaining both the eastern and western flow branches actually causes worsens flooding around Ashby Ave and downstream due to coinciding flow peaks. While some upstream detention helps mitigate flooding issues in the study area, proposing large detention facilities along both flow branches is not recommended.

Results show the alternative utilizing detention in both DA1 and DA2, with infrastructure improvements, provides the most improved flood mitigation. During the 2-year, 24-hour storm, no buildings are flooded under this alternative scenario, compared to three (Houses C, D, and E) flooded under existing conditions. Road flooding on Clinton St, Suter St, and Ashby St are also eliminated, flooding depths and extents in the yard area upstream of Suter St are significantly reduced, and flow is confined within the channelized path through the open space upstream of Ashby Ave. Three out of the seven buildings analyzed that experience flooding under existing 10-year storm conditions were longer flooded. Moreover, for the September 1st storm, this alternative eliminated flooding at two of the eight analyzed locations with existing flood concerns. Overall, 10-year, 24-hour storm and September 1st storm flood depths were the lowest during this alternative.

Alternative		Results		
		2-year, 24-hour Design Storm	**10-year, 24-hour Design Storm**	**Sept. 1, 2021 Storm Event**
3- COMBINATION (DETENTION + INFRASTRUCTURE)	**A**	Lowers all flood WSE's below building FFE's, eliminates all 3 flooded buildings previously measured at existing conditions and eliminates abutting floodwater from 7 buildings previously measured at existing conditions. Significantly reduces planar flood extents in open space areas and eliminates flooding along the roads.	Significantly lowers all flood WSE's, eliminates FFE flooding of 3 of 7 buildings flooded during existing conditions and eliminates abutting floodwater from 4 buildings previously measured at existing conditions. Significantly reduces planar flood extents in open space areas and along the roads.	Significantly lowers all flood WSE's, eliminates FFE flooding of 2 of 8 buildings flooded during existing conditions and eliminates abutting floodwater from 2 buildings previously measured at existing conditions. Significantly reduces planar flood extents in open space areas and along the roads.

5.0 Cost Estimate

Below is a summary of each of the alternatives. A detailed breakdown of the cost estimate is in Appendix I.

Charles St to Washington St Drainage Improvement Study
City of Harrisonburg, VA
September 23, 2022

Preliminary Cost Estimate of Alternatives						
Pay Item No.	Alt. 1A Cost	Alt. 1B Cost	Alt. 1C Cost	Alt. 1D Cost	Alt. 2B Cost	Alt. 3 Cost
Detention Facility						
	$ 3,847,500.00	**$ 2,995,000.00**	**$ 6,997,500.00**	**$ 10,730,000.00**	**$ -**	**$ 6,997,500.00**
Storm Infrastructure						
	$ -	$ -	$ -	$ -	**$ 1,627,315.00**	**$ 1,627,315.00**
Road Reconstruction						
	$ -	**$ -**	**$ -**	**$ -**	**$ 52,507.50**	**$ 52,507.50**
Re-Seeding						
	$ -	**$ -**	**$ -**	**$ -**	**$ 625.00**	**$ 625.00**
	$ -	$ -	$ -	$ -	$ 625.00	$ 625.00
	$ 3,847,500.00	**$ 2,995,000.00**	**$ 6,997,500.00**	**$ 10,730,000.00**	**$ 1,680,447.50**	**$ 8,677,947.50**
Assumed Contingency Items Associated to Subtotal Costs						
	$ 3,031,987.50	$ 1,512,475.00	$ 3,533,737.50	$ 5,418,650.00	$ 848,625.99	$ 4,382,363.49
Project Subtotal						
	$ 6,879,487.50	$ 4,507,475.00	$ 10,531,237.50	$ 16,148,650.00	$ 2,529,073.49	$ 13,060,310.99
	$ 1,719,871.88	$ 1,126,868.75	$ 2,632,809.38	$ 4,037,162.50	$ 632,268.37	$ 3,265,077.75
	$ 8,599,359.38	**$ 5,634,343.75**	**$ 13,164,046.88**	**$ 20,185,812.50**	**$ 3,161,341.86**	**$ 16,325,388.73**

6.0 Summary and Recommendations

Based on the analysis and results, there is no single best solution to the completely eliminate home flooding from occurring in the study area. The existing roadway infrastructure contributes to the backwater-controlled flooding that occurs in the study area. Within the topographic and existing infrastructure constraints, flooding cannot be eliminated from all yards, roads, or more importantly, impacted buildings adjacent to the channels, especially during lower frequency storms such as those comparable to the 10-year, 24-hour storm or greater. Moreover, while detention facilities help decrease the flows traveling through the study area, because detention can alter the timing of flow peaks, any potential SWM management facilities proposed upstream of or within the study area must be carefully designed to ensure no detrimental impact to downstream flow peaks.

Based on the H&H analysis results and the cost estimate, Alternative 2B, storm sewer infrastructure improvements (maximizing culvert capacities and adding rigid channels through the open areas), is the best alternative to show reductions in both flood depths and lateral flood limits while also considered the most cost-effective of the alternatives assessed, as well as not causing detrimental downstream flow impacts. Building flooding can be significantly reduced for all but one of the evaluated buildings flooded under existing conditions during the 2-year storm. Flood depths are all lowered relative to existing conditions, even if still above first floor elevations at certain locations, during the 10-year storm and the September 1st storm. Prior to implementation, additional costs and construction impacts on the community should also be considered.